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THE MECHANICS OF

ROCKFILL CONSOLIDATION

A THESIS

Presented to

The Faculty of the Graduate Division

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ROCKFILL CONSOLIDATION

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SUMMARY

The purpose of the research herein summarized was to define more clearly the mechanics of rockfill consolidation and to correlate the results from this laboratory testing to measurements on actual rockfills. The experimental work consisted of three long-term consolidation tests on uniform sized samples 7.5 inches in diameter and roughly 4.0 inches high. The first test was performed on granite and tests 2 and 3 on a metamorphosed feldspathic sandstone called greywacke. The individual particles were between 1.0 and 1.5 inches in diameter. The loads on the samples ranged between 1,634 and 32,700 pounds per square foot.

The main purpose of this endeavor was satisfied in that a confirmation of the suspected mechanics of rockfill consolidation was obtained. The settlement of rockfills is due to the spalling and breaking of the points of contact of the individual particles within the fill and any finer material trapped between these points. The settlement of the lab samples after each increment of loading was similar to that of an actual rockfill in that the settlement continued at a constantly diminishing rate.

While testing the first two samples, considerable crushing of the top and bottom layers of rock was observed. In the third test, two epoxy caps were used on the sample to obviate the flat bearing surface and significant differences were noted between the results of tests 2 and 3.

From a comparison of tests 2 and 3 it was deduced that when a rockfill material is tested using normal loading procedures (no cap), the results do not reflect the action within the actual fill. The capped sample, however, settled with no sudden movement and simulated the action of a larger fill more closely. It was also noticed that local settlement is more prevalent when the simulated fill is made of flat, elongated pieces of broken rock. This local settlement appears to be unavoidable.

An effort was made to check the supposed benefits derived from sluicing a fill during construction. It was found in this testing that sluicing is beneficial in reducing the amount of settlement after the construction of a rockfill by lowering the rock's unconfined compressive strength to its minimum value during construction. The physical characteristics of the rock itself were found to play an important role in its resistance to settling.

This research suggests that although it was not possible to determine many of the significant properties of rockfills by these laboratory consolidation tests, it should by no means, be the end of laboratory testing of rockfills. Experiments should be undertaken to observe directly the interior of a rockfill during construction and subsequent application of water load.

CHAPTER I

INTRODUCTION

Before we can state clearly the mechanics of rockfill consolidation, we must define the term "rockfill." In this paper, rockfill means that the fill is made of rock in which at least half of the material in its maximum cross-section is comprised of loose rock placed by dumping. This is consistent with engineering literature on this subject (3). Dumping means any method of placing other than by hand.

In normal consolidation testing of saturated clays, silts and other fine-grained impervious materials, the Terzaghi theory is used to describe the action taking place.

In essence, this theory states that in any sample prior to testing, the sample has an initial void ratio e_1 and an initial pressure on the sample p_1. When a pressure p_2 is placed on the sample, some consolidation may take place depending on the amount of air forced out of the sample, compressed within the soil and forced into solution. In a saturated sample this settlement is usually quite small due to the water within the voids carrying the change in pressure, Figure 1.

As the water seeps from the sample (at a rate proportional to the sample's permeability), the load is gradually assumed by the grains themselves and eventually the pore-water pressures will dissipate leaving the soil skeleton to carry the new pressure p_2. This immediately brings to light the great difference between normal consolidation testing and that of rockfill material. The assumptions of Terzaghi

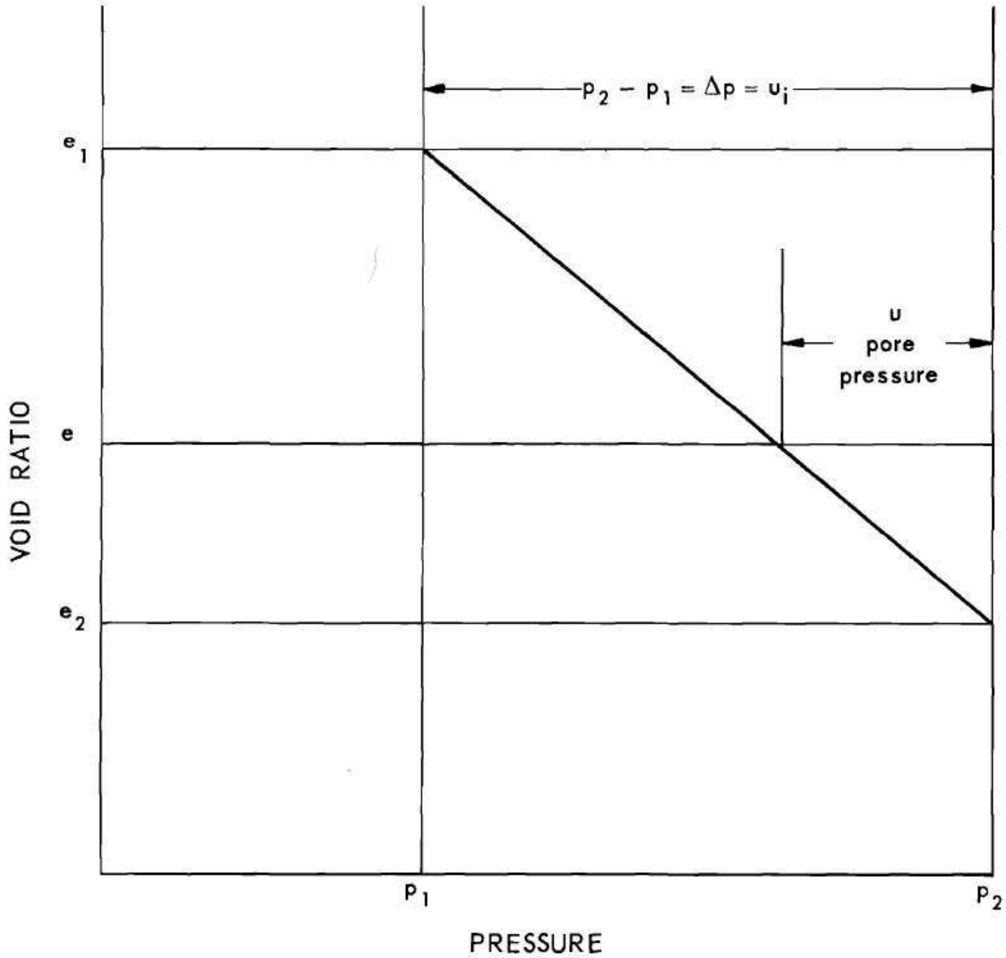


Figure 1. Pressure-versus-Void Ratio Relationship for a Pressure Increment (Terzaghi's Theory).

specifying (1) a homogeneous soil, (2) complete saturation, (3) negligible compressibility of soil grains and water and (4) the greatly idealized pressure-versus-void-ratio relationship shown in Figure 1 are just a few of the reasons which obviate any realistic application of this theory to the time-rate of consolidation tests on rockfill. A theory or empirical means by which total settlement of rockfills could be predicted also does not exist.

The explanation for rockfill consolidation up to the present time has been largely due to intuitive reasoning of how a body of this type could possible settle. It is usually assumed that the points of contact within the mass break, settlement takes place rather rapidly with a reorientation of the particles and that no appreciable settlement is further noted. Little investigating has been done to verify this reasoning or to relate laboratory testing of rockfill to actual settlement records (7). The purpose of this research is to make a step in this direction and perhaps give a clue toward developing a theory which envelopes this phenomena.

A factor which influences the amount of settlement to a large degree is the presence of fines (or finer material than the rocks themselves) between the points of contact. Obviously, in the process of consolidation these smaller pieces will break first causing considerable settlement and then fall into the voids of the fill allowing the larger pieces to contact each other. This problem did not exist in this testing due to the fact that only clean material of uniform size was used. This does, however, make difficult any attempt to compare percentages of the settlement or other related data between actual fills and laboratory test models. This is especially true if the fill was not properly sluiced during construction.

Sluicing is a construction operation performed on the rockfill as it is being built. The normal process is to direct jets of water in large quantities and under relatively high pressure upon and against the rock as it is being dumped. This is also done on the face of the fill between dumpings.

There seem to be at least two, if not three, different viewpoints on the benefit or the manner in which benefit is derived from sluicing. Some say that the purpose is to cause the pieces of rock to come to rest more nearly in a final position than would otherwise be the case. At the same time the sluicing removes the fines from between the points of contact and thereby reduces the settlement in this manner. Terzaghi (1) suggests that since in most sluicing operations the mechanical action of the jets discharged by the monitors is limited to the uppermost portion of the slope and extending no further than 30 or 40 feet down the slope that the benefit derived from sluicing is actually due to the water softening of the rock.

Terzaghi explains that the water saturates the outer, more desiccated portions of the rock where the corner breakage occurs and this saturation reduces the compressive strength of the rock to its minimum value. This then allows a greater amount of settlement to take place during construction and reduces the settlement which takes place afterwards.

The weakening influence of saturation on the compressive strength of rock has been known for many years (2). The ratio q'_d/q_d between the strength q'_d of the saturated rock and q_d of the dry rock is called the coefficient of softening N_s. The following data were published more than 40 years ago (2):

Sound granite, Austria	q_d = average 21,000 PSI
	N_s = average 0.88
Granites from Sweden and Germany	q_d = average 35,000 PSI
	N_s = average 0.94
Crystalline Limestone	q_d = average 14,000 PSI
	N_s = average 0.90

CHAPTER II

EQUIPMENT AND PROCEDURE

Equipment

The apparatus used in this test on rockfill did not vary greatly from that of normal consolidation tests on fine soils. The necessary apparatus consisted of: (1) a suitable container for the sample (consolidometer), (2) a loading device which was capable of maintaining a constant load on the sample regardless of the amount or rate of deflection, and (3) an accurate means of measuring this deflection.

The consolidometer for this testing was made of seamless steel tubing 7.5 inches in inside diameter with a wall thickness of 0.5 inches. The base and loading head were machined from 1.5 inch steel plate. A coefficient of lateral earth pressure of one was assumed for the design of this piece of apparatus because the exact lateral pressures could not be predicted and the future use of this consolidometer might include consolidation testing of clays with higher coefficients of lateral earth pressure than rockfills. As in most laboratory equipment this obvious overdesigning was done to reduce the deflections of the consolidometer to a negligible amount.

For a loading device, this writer was fortunate enough to have access to a piece of equipment especially developed for consolidation testing -- the Conbel loading device, Figure 2. This machine has been designed for consolidation testing loads on normal size samples or for

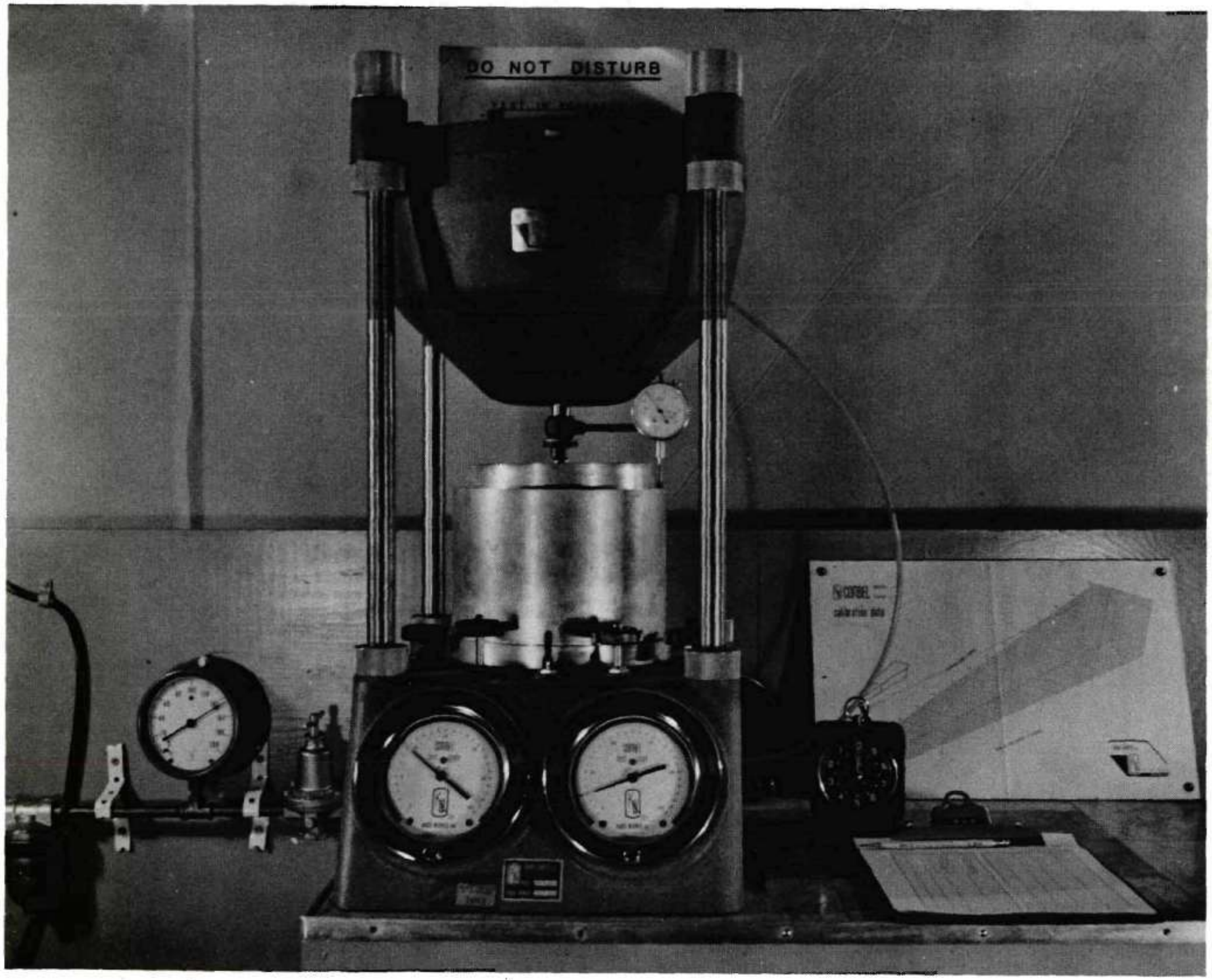


Figure 2. The Conbel Loading Device.

normal loading on samples of such size as used in this work. This machine has the ability to maintain a constant load regardless of the amount of sample deflection.

The major control components of this instrument are: (1) an enclosed piston-diaphragm system, (2) precision air regulators, (3) pressure gages and (4) control valves.

The air regulators control the air pressure acting on the piston and, consequently, the load applied to the sample. A continuous supply of dry air is required at 125 pounds per square inch pressure.

The pressure gages at the front of the instrument are designed with a repeatability of between 1/4 to 1/2 of one per cent. They are equipped with a reflector strip to eliminate parallax in reading. These gages represent two complete regulating and recording circuits which enable the use of low loads with good sensitivity (this was found to be true only if the high-range air regulator was left partially open to minimize air loss). A low-range shut-off valve is provided to remove the low circuit from operation at the higher loads, and a push-pull type air switch to change from one range to another. The machine has a spring rate of approximately nine pounds per inch of piston travel which, for all practical purposes, is negligible.

The deflection was measured by a micrometer attached to the ram as shown in Figure 2. This micrometer could be read directly to one-thousandths of an inch and estimated to ten-thousandths of an inch.

Procedure

The sample for the first test was granite, the choice being based for the most part on convenience since granite is prevalent in this area and is frequently used in construction. This sample, besides being a "pilot" test, also served as a comparison between dissimilar rock types for tests 2 and 3 were performed on metamorphosed material.

In addition to defining the mechanics of rockfill consolidation, it was thought that at the same time this research might be correlated to the settlement data from actual rockfills. To accomplish this, samples were obtained from Nantahala Dam in southwestern North Carolina.

Nantahala Dam is a 255 foot high sloping core rockfill dam from which settlement data have been obtained monthly since February, 1942, shortly after the completion of the dam.

Material for the entire dam was obtained at the site and the rock used was a metamorphosed feldspathic sandstone known as greywacke. This material has an unconfined compressive strength of approximately 25,000 pounds per square inch. Tests 2 and 3 were performed on this material.

<u>Testing Program</u>

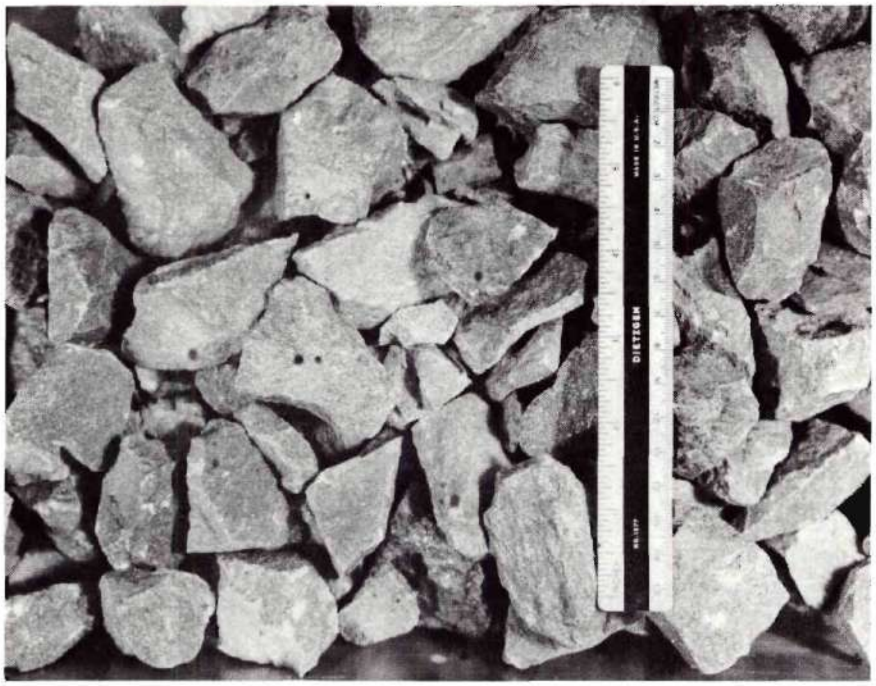
The testing program consisted of three long-term consolidation tests on uniformly sized samples 7.5 inches in diameter and roughly 4 inches high. The first test was performed on granite and test nos. 2 and 3 on the greywacke.

<u>Preparation of Sample</u>. In each of the three tests performed, the sizes of the rock fragments were kept approximately the same. The test samples were obtained from the rocks passing a 1.5 inch sieve and retained on a 1.0 inch sieve. This made the largest individual rock particle roughly one-fifth of the diameter of the consolidometer. No

attempt was made to select rocks with similar shapes. The Nantahala material was obtained directly from the dam and was chosen as near the desired size (1.0 to 1.5 inches) as possible so that no crushing of larger stones was necessary. The greywacke was, therefore, a quarried stone. The granite was obtained from a local granite dealer and was also a quarry-run material very near the size desired. By sieving, the necessary amounts of each type material were obtained for the models. The granite was sub-rounded and the greywacke was more angular as can be seen in Figure 3.

The method of placement was identical in test no. one and two. Once the proper size material was obtained, it was dropped in the consolidometer and raked by hand to insure that there were no large voids within the fill and that each rock was seated in place. Since the validity of comparing a sample placed in this manner to that of field placement might be questioned, it might be of interest to note that, in essence, the sluicing process under which most rockfills are placed accomplishes almost the same end result by removing the fines from the points of contact of the larger stones. This, in turn, seats the particles and reduces the amount of settling within the fill. The fines that settle into the voids of the fill usually do not affect the fill's resistance to settlement or its permeability to any great extent (1), but this, of course, depends on the proportion of fines within the fill.

A special effort was made, however, to turn the topmost stones in such a manner so as to place as large a bearing area as possible upon which to rest the loading head. This was done because this writer thought that in doing so the action of consolidation in the laboratory



a. Granite Sample After Testing

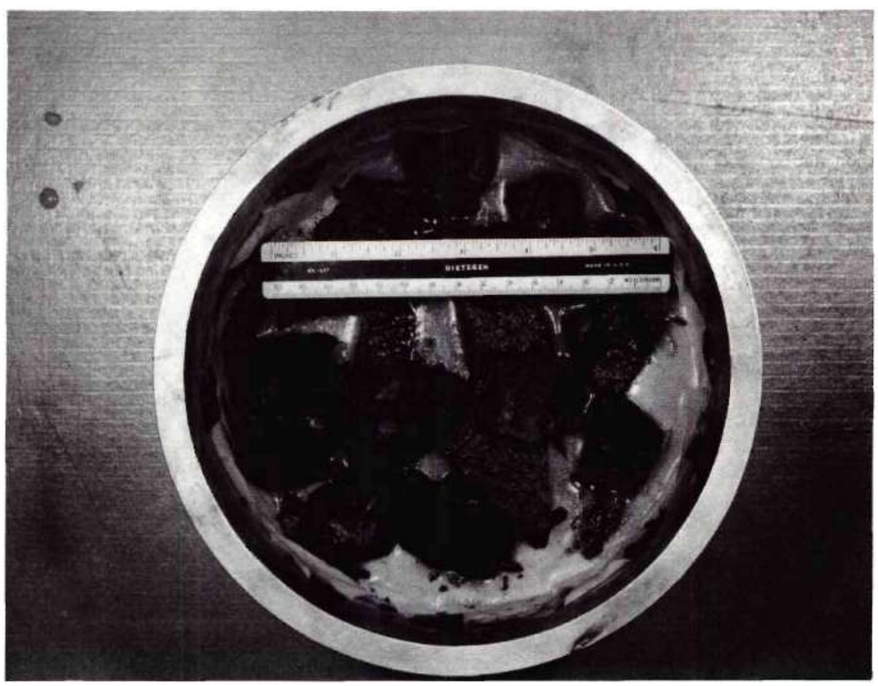
b. Nantahala Greywacke After Testing

Figure 3. Photographs a. Granite Sample After Testing and b. Nantahala Greywacke After Testing.

test would more closely resemble consolidation in larger fills. In test 3, however, this procedure for the construction of a test model was somewhat changed. In this test an effort was made to reduce the local crushing at the points of contact of the top and bottom layers of rocks by the use of epoxy caps at these two surfaces, Figure 4.

The procedure for making these caps was relatively simple. Teflon film was used to protect the metal consolidometer from the action of the epoxy. After a layer of this film had been spread upon the base of the consolidometer, a thin layer (approximately 1/2 inch) of epoxy was poured into the bottom of the consolidometer. As this epoxy began to harden, the rock sample was placed in the consolidometer in the same manner as test samples one and two. Time was allowed (two days) for this base cap to harden before the top cap was placed. The procedure for making the top cap, however, was slightly different in that the epoxy did not bond to the rock itself. Again a Teflon lining was used around the edge of the consolidometer to protect it from the epoxy but in this case a thin plastic film was provided for the top layer of rock to prevent the epoxy from seeping through the sample. In essence, the top cap was a "form-fitted" one. The thin plastic film (Saran Wrap) was placed and contoured over each individual rock at the surface of the sample. The epoxy was then poured on this surface and allowed to harden. The benefit expected to be derived from this was a reduction in the amount of crushing of these top and bottom stones. The role of the cap in this effort was to distribute the applied load more evenly over the surface of the top and bottom layers of stones rather than concentrating it at the corners as the flat loading head

a. Epoxy Cap, Consolidometer and Greywacke After Test 3



b. Epoxy Base

Figure 4. Photographs a. Epoxy Cap, Consolidometer and Greywacke After Test 3 and b. Epoxy Base.

did in tests 1 and 2.

The choice of a suitable material for this "cap" was not an easy problem to be solved. An ideal material for this job would be the one which has before setting an infinitely large modulus of elasticity of 900,000 pounds per square inch and an unconfined compressive strength of approximately 6,000 pounds per square inch. For all practical purposes, the deflections of the cap and the plastic film were negligible for all increments of loads used.

Test 3 was different in another respect. While under the action of the 5,000 pound load and near the end of settling under this load (as deduced from the time-settlement curve) the sample was flooded and allowed to soak for about 20 minutes before draining. Since saturation reduces the compressive strength of rock, this flooding was performed to see if this could be one of the real benefits of sluicing in large rockfills.

Loading Procedure and Testing. From this point on the testing procedure followed very closely to that of normal consolidation testing, the exception being that the loads, of course, are much smaller in normal testing. The first increment was chosen as 500 pounds (1632 psf) and this was roughly doubled for each following increment of loading up to the highest load of 10,000 pounds (32,600 psf). Under each increment of loading, sufficient data was taken so that a time-settlement curve could be plotted (see appendix).

In testing the Nantahala greywacke, the criterion for determining when the settlement was complete for a certain applied load was that the sample must not have settled one ten-thousandths of an inch in at least 36 hours. This was considered 100 per cent consolidation. On this basis when the consolidation was complete under the last load increment, the sample was removed and checked for the change in gradation (see appendix). The specific gravity of the rock (G_s) was determined and the initial void ratio was determined from the measurement of the sample.

The coefficient of softening, N_s, was found for the greywacke by running several unconfined compression tests on both soaked and dry samples two inches high and seven-eighths of an inch in diameter. The soaked samples were not fully saturated but contained what moisture they could by soaking in water for one hour. This method was apparently effective in moistening the sample for when the saturated cores failed, the inner material was dark from moisture. The coefficient of softening of the greywacke was found to be .90 using this procedure.

CHAPTER III

DISCUSSION OF RESULTS

Mechanics of Consolidation

When the granite was placed under the first increment of load (1,634 pounds per square foot) in test 1, a sudden settlement was observed accompanied by loud snapping sounds. Within a minute or so after loading, the noises ceased and the rate of settling began to fall off. From this point on no further snapping could be detected within the model and the settlement continued at a constantly decreasing rate.

This phenomenon was repeated again in subsequent load applications including the final load of 32,700 pounds per square foot. The only differences noticed in the higher loads were that the noises during the first few moments after loading were louder and that they continued over a longer period of time with increasing load.

In test 2 the same cracking was noticed but it was not the sound of an extremely brittle material as was heard in the granite model. The cracking of particles could be heard from time to time throughout the first hour or so after each load increment which indicated that some of the cementation provided by nature in the greywacke was breaking down and allowing further settlement.

A very odd looking time-settlement curve was obtained from the data of test 2. In the latter portion of the curve for each increment of load, the sample underwent a sudden increase in both the amount and rate of settlement. This was faintly evident in the 1,634 pound per square foot load and was magnified at the higher loads, Figure 5b. The reason for this was the flat loading head and base used in test 2, because in test 3 using the epoxy caps, no evidence of a sudden settlement or change in rate was noticed, Figure 6b.

During test 3 in the final stages of loading (the 5,000 pound increment), the sample was flooded and further settlement immediately took place. The rate of settlement increased gradually until the water was drained from the sample some 30 minutes after its induction. This roughly simulated the wetting that sluicing affords. The rate of settlement at this time slowly began to reduce. The sample settled thirty-seven thousandths of an inch more than the settlement due to the 5,000 pound load up to the time of flooding. This represents an increase in settlement of 45 per cent which would indicate that sluicing is of considerable benefit in reducing the amount of settlement in rockfill by softening the contact points.

The characteristic shape of the time-settlement curve of a rockfill is quite different than that of a finer more impervious soil. The initial settlement on rockfill is relatively large with the settling continuing at a constantly decreasing rate.

Rock Type

The difference in settlement potential of the types of rock used in this testing is readily apparent when viewing Figure 5. The total settlement was less using granite as was expected (due to its high compressive strength) but the curve was also smoother than the time-settlement curve of a comparable load in test 2. In all fairness to this comparison, it must be said that the greywacke is probably more likely to have a broken-line time-settlement curve because of the method of testing used in tests 1 and 2. The graywacke, a metamorphosed rock, has its cementing materials only partially or incompletely recrystallized. Its strength is, therefore, less than that of completely recrystallized (metamorphosed) rock or of a heterogeneously arranged aggregate of crystal grains that one finds in an igneous rock. Under loading, this incompletely recrystallized cement may be expected to rupture at a lower stress than thoroughly recrystallized materials. Under continual load, this cementation seemed to rupture allowing the sample to settle, stop and settle again after further grain-to-grain contacts were broken. This was noticeable at the surface and the bottom of the sample where the flat edge of the loading head and base plate crushed the rocks to a fine powder. To bear this out, each of the loading increments using the epoxy caps was smoother and straighter than its counterpart in test 2, Figure 6. This smoothness is definitely attributed to the absence of the crushing of the samples at the surface and bottom and gives a much more realistic picture of the rocks' behavior within a large fill.

The only evidence of other consolidation testing of rockfill points out the difficulty in correlating data from laboratory tests performed with flat bearing areas for loading the sample at the top and bottom. In order to check the adequacy of the rock to be used in Ambuklao Dam, a series of crushing tests were made. These tests were concentrated on a particular metamorphic rock to be used (not named) and a dioritic rock. The crushing tests were performed by placing

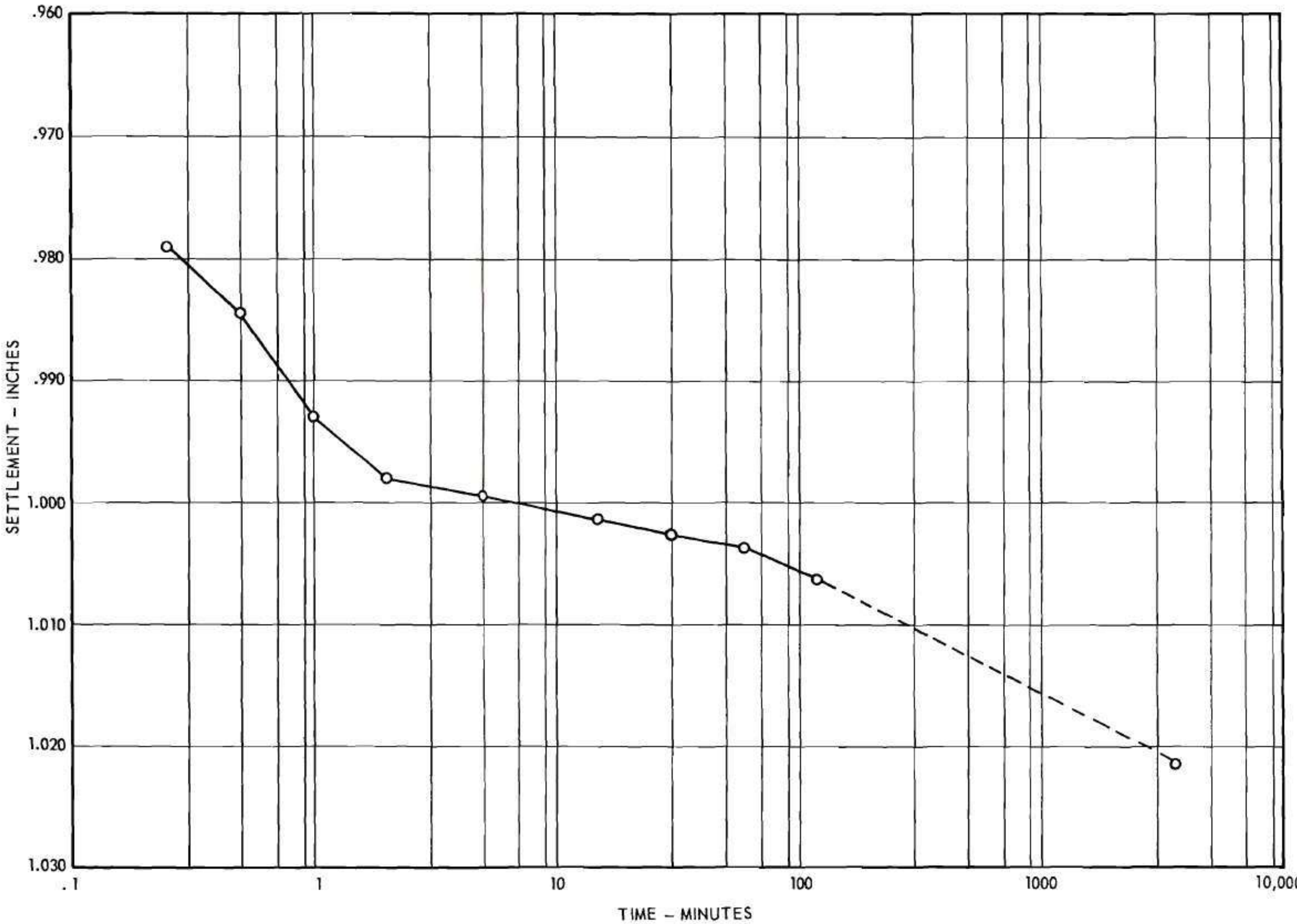


Figure 5a. Time-Settlement Curve for 32,700 psf Load on Granite in Test 1.

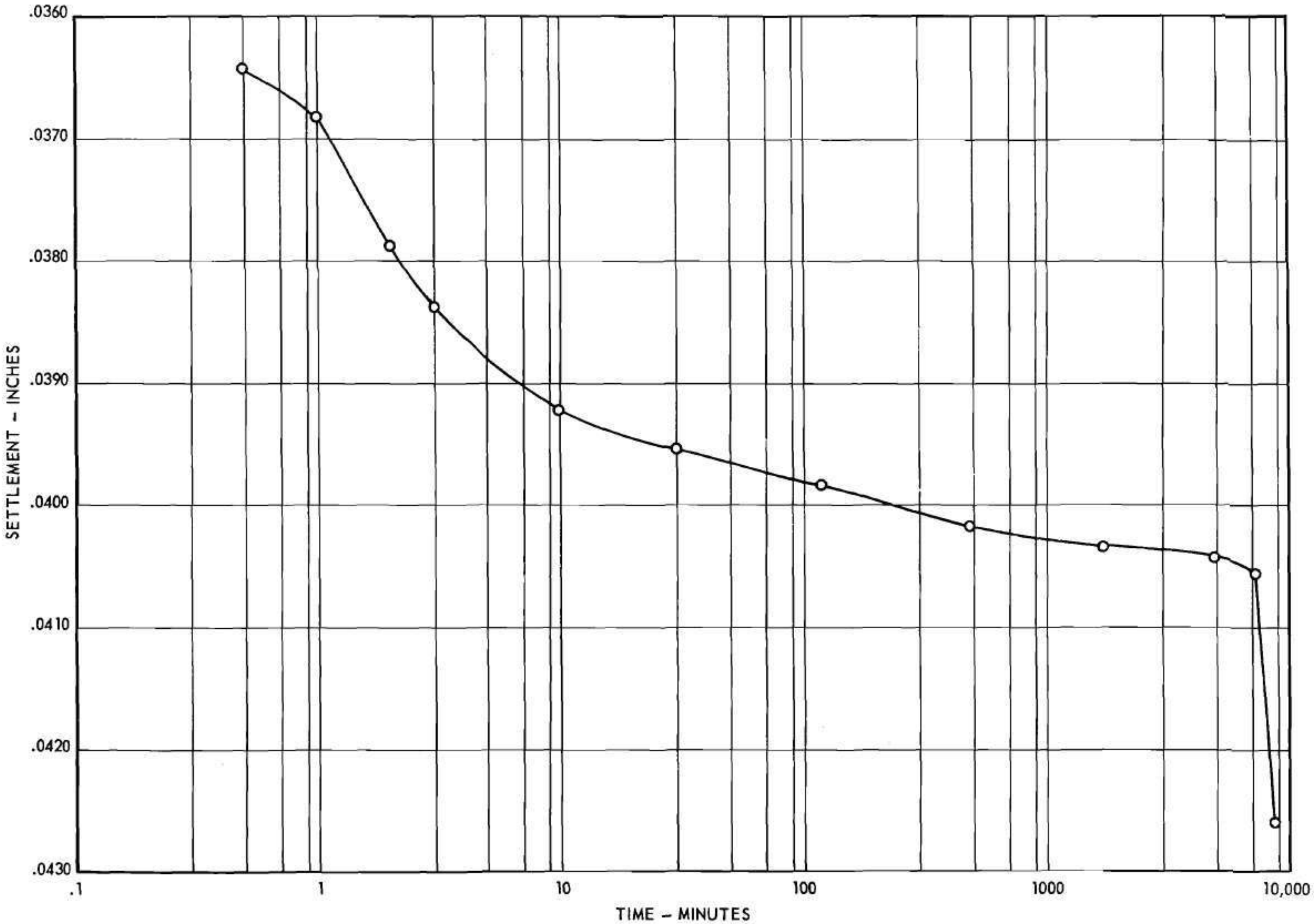


Figure 5b. Time-Settlement Curve for 32,700 psf Load on Greywacke in Test 2.

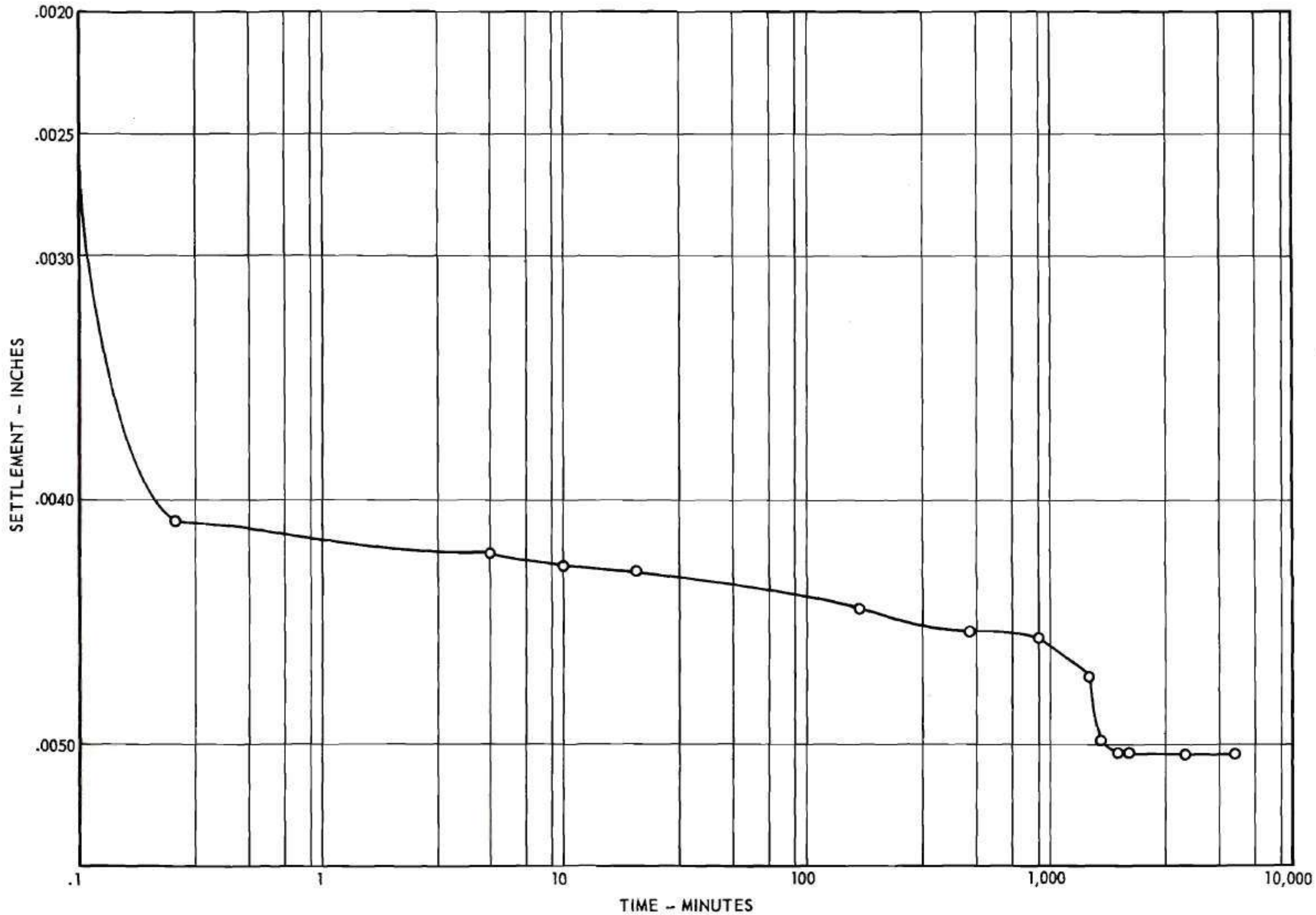


Figure 6a. Time-Settlement Curve for 3264 psf Load on Greywacke in Test 2.

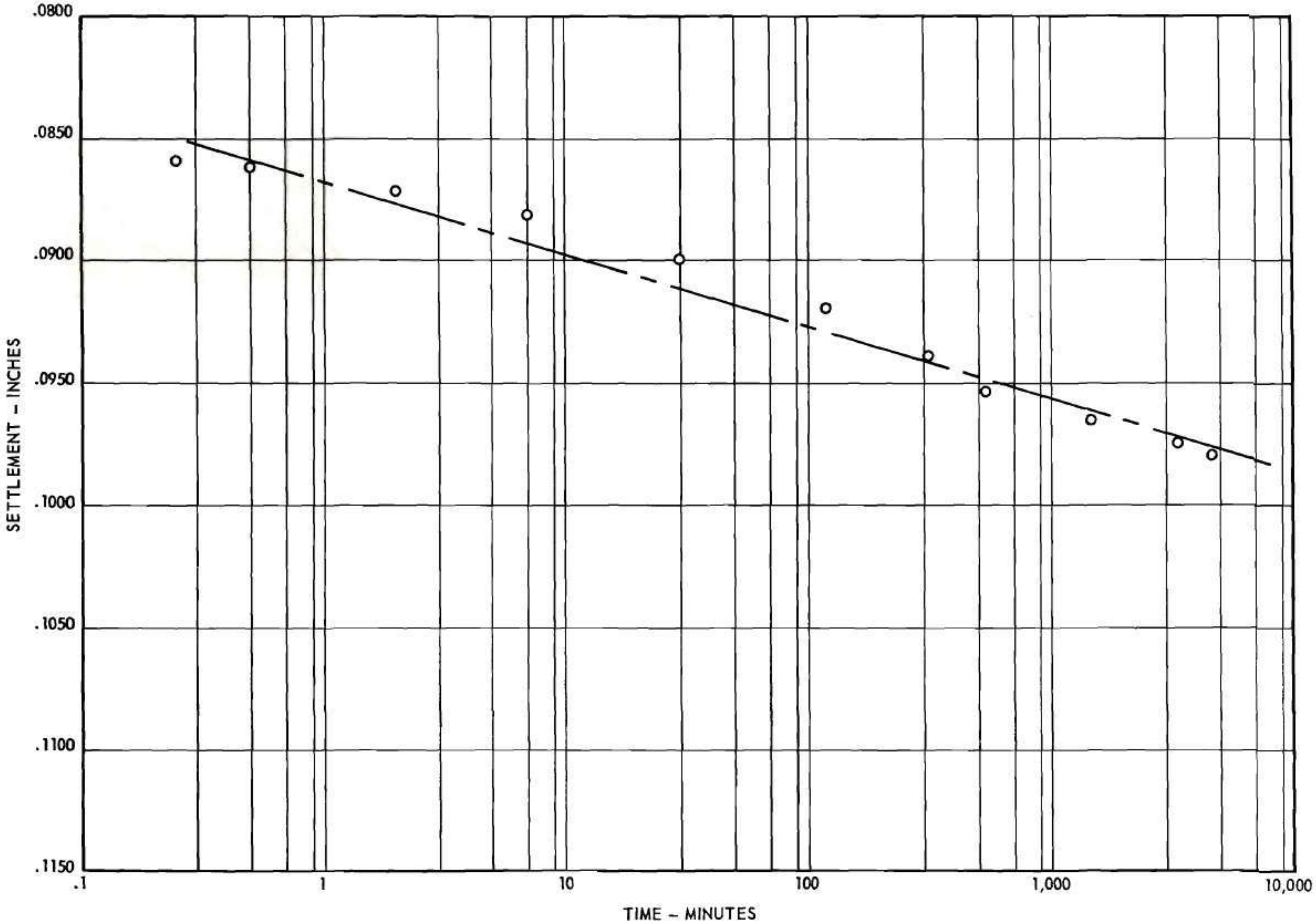


Figure 6b. Time-Settlement Curve for 3264 psf Load on Greywacke in Test 3.

samples of the broken rock from the quarry areas in a container and loading the surface of the sample with a load equal to the pressure that would be exerted on the bottom layer of the rockfill by the overlying fill material. The container was a 30-inch-diameter cylinder 20 inches deep. A hydraulic jack and a loading plate 20 inches square were used to develop a pressure of approximately 25 tons per square foot on the material in the box. The settlement under the load shown in Figure 7 indicated how much the rock might break down under the pressures to be expected in the dam,or so they thought. Fucik points out that prior to this testing the general expectations were that the diorite would prove to be much stronger than the metamorphic rock. The results were so contrary to what had been expected by visual inspection, however, that at first it was suspected that the results had been reversed. The truth was that the diorite was more damaged by blasting and, hence, settled more due to the numerous fines in the sample. Something that Fucik did not consider is that the diorite might have been more susceptible to the local crushing action of the flat loading head and that no realistic picture of how a particular rock will perform in a fill can be determined by testing in this manner.

Regardless of the differences in rock type and method of testing of the models under discussion here, it has been found that a general statement can be made in regard to the mechanics of the consolidation of these models. The particles within the fill undoubtedly have irregular surfaces and edges which come to rest against each other in the process of dumping. The increase of load causes these edges or points of contact to spall or crush and, hence, there is some readjustment within the fill. This process continues until the contact points have become sufficiently blunt to transmit the load to one another without further spalling or crushing. This phenomenon coupled with the further crushing of the trapped material between the points of contact leads to a resulting settlement continuing at a gradually diminishing rate. This is true in the laboratory tests of this work and in actual measurements on large rockfills, mainly rockfill dams, Figure 8.

Comparison of Data to Actual Rockfills. To compare or relate the results from three laboratory tests to actual rockfills is very difficult for the variables and uncertainties are many. In actual rockfills, because of the segregation, the absence of beneficial sluicing past the uppermost part of each layer of rock and the effects of corner breakage in a downward direction, every lift represents a system composed of zones with different compression characteristics. In a dumped rockfill made out of a succession of lifts, the compression characteristics of the fill change at the level of the top of each lift. In this complicated picture, though, there are certain facts which have come to light from this testing to point out that all similarity between lab models and actual fills is not lost and that it might be possible through principles of similitude to relate the actual data to the laboratory data. Natahala Dam has settled 1.2 per cent which compares relatively well with the results of test 2 which settled 1.4 per cent under comparable loads. This may or may not be indicative but only further testing can bear this out. It might be possible to obtain a rough idea of the amount of settlement to expect from a dam by

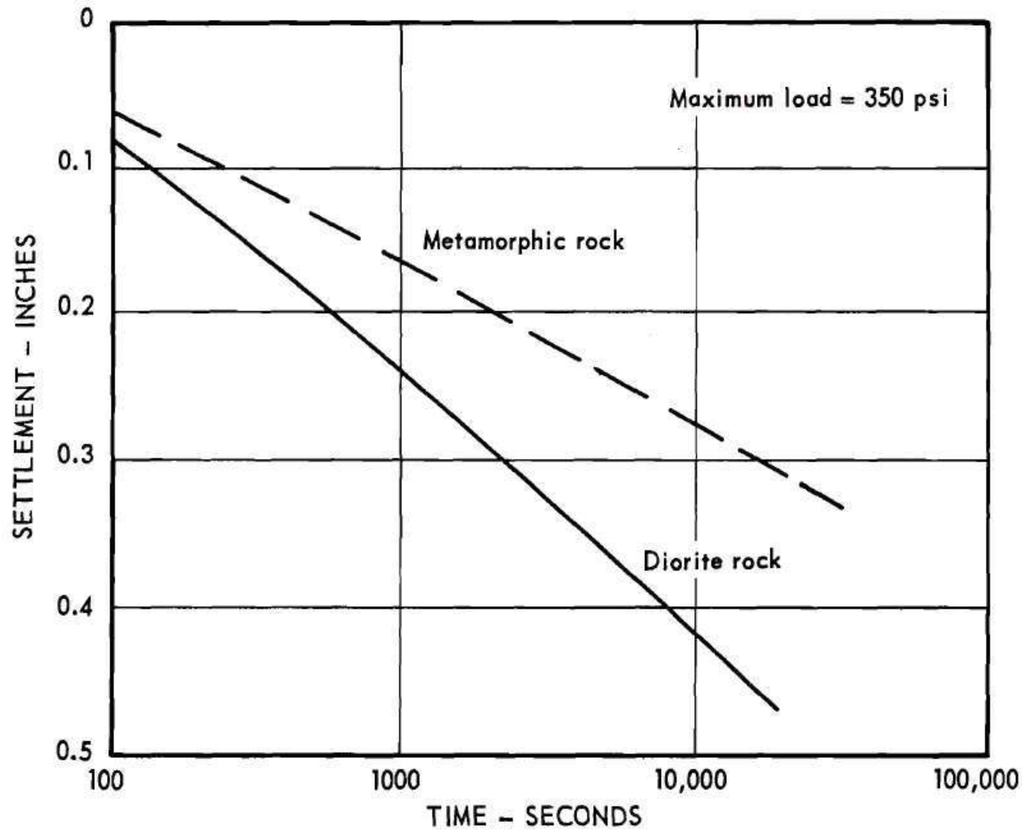


Figure 7. Settlement Curves from Crushing Tests Performed on Materials from Ambuklao Dam.

performing a test similar to the ones of this work on the rockfill material.

In general, the settlements of the models can be said to be realistic although they extended over a much longer period of time than was originally estimated. Due to the small model, it was thought that one day per increment of load would easily bring about all noticeable settlement but this was not the case. Each load increment required at least three days and some as much as seven days, the higher loads requiring the longest time. This long-term settlement is also noted on large rockfill dams such as the Dix River Dam which has been settling for the past 40 years and judging from the shape of the settlement curves will continue to settle for the next 10 to 20 years.

An interesting point to note is shown in Figure 9. With the exception of the 10,000 pound load curve, 97 per cent of the total settlement for each increment of loading took place within 30 hours. It so happens that the 10,000 pound curve would have also met this requirement had not the sample been jarred during the last part of the settlement curve causing the total settlement to change. In other words, this curve probably would have been shifted down on the graph and would have been comparable to the other curves had it not been disturbed. It was noticed that the sample from test 2 was much more sensitive to shock in the latter portions of each time-settlement curve than the sample which was capped. This was apparently also due to the settlement mechanism in contact with the base and top of the sample as previously described.

Immediately after applying each increment of load, it was noticed that the settling process was extremely sensitive to shock. The slightest

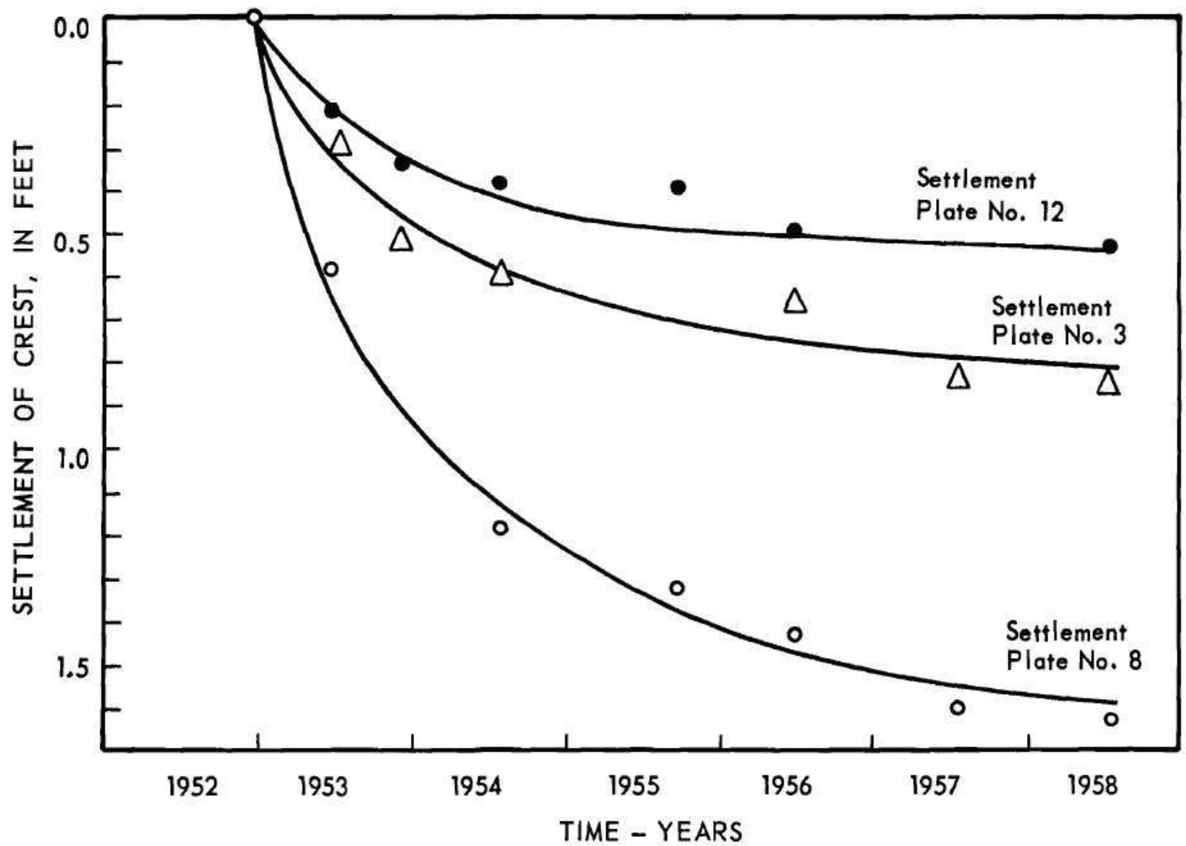


Figure 8. Characteristic Settlement Curves for Kenney Dam.

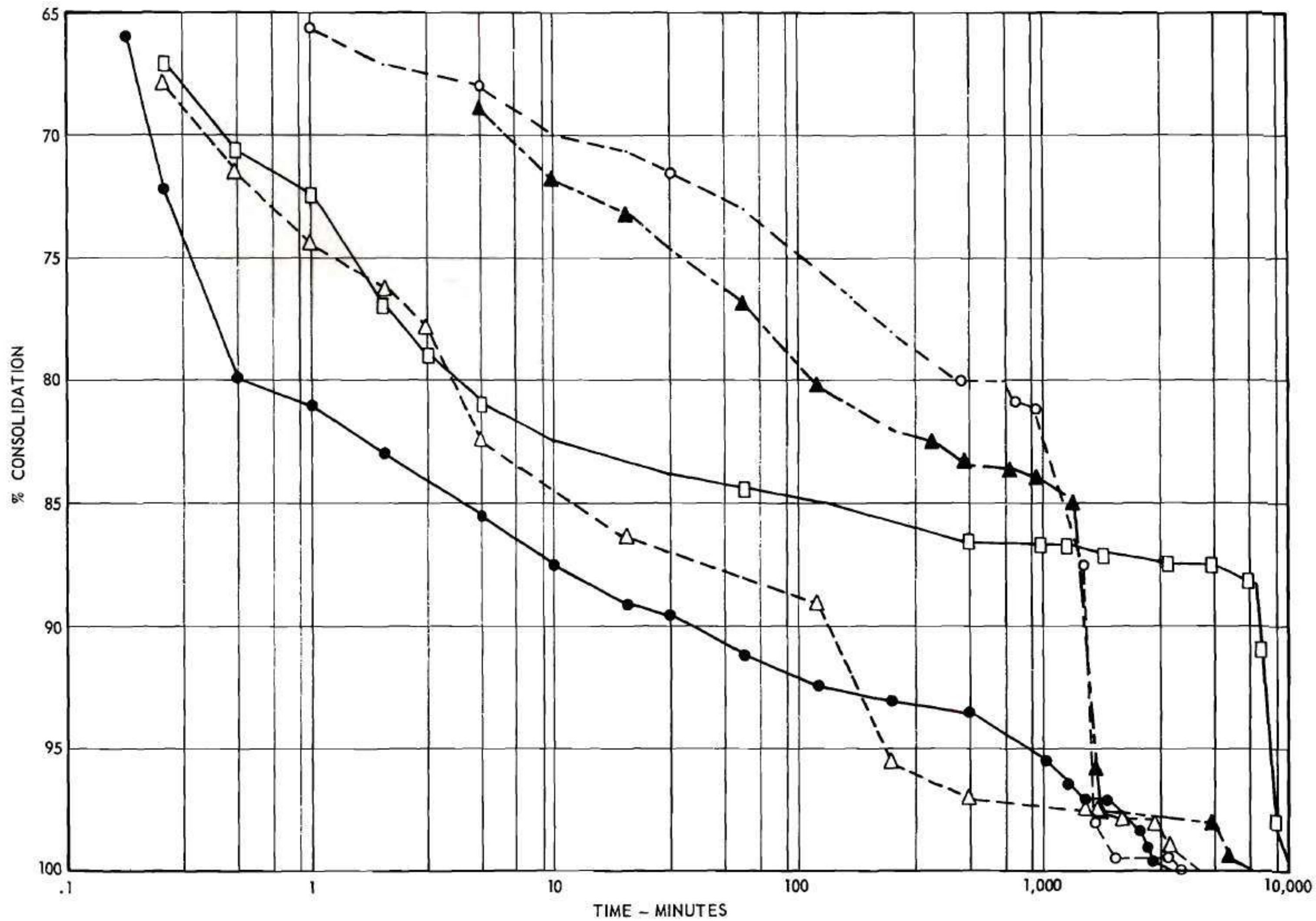


Figure 9. Relationship Between Time-Per Cent Consolidation Curves of Increments in Test 2.

jar in the early stages of each load application produced a definite increase in settlement. This sensitivity gradually decreased, especially with the capped samples due to the fact that the particles found a more stable position sooner. This inherent characteristic that rockfills have has been a much discussed topic, especially where rockfill dams have been proposed in known "earthquake-prone" areas. It seems, though, that the action of the laboratory sample is comparable to actual rockfills in this respect also. Shortly before midnight on August 17, 1959, a severe earthquake occurred in the Hebgen Dam area in Montana. The epi-center of this quake was reported to have been at a point approximately 300 miles from Brownlee Dam. Intensity at the epi-center was variously estimated at 7.2 to 7.8 on the Richter scale. On August 18 readings were taken on all monuments and settlement rods along the crest of Brownlee Dam. Those readings did not reveal any effect of the earthquake on the rate of movement. Since this was one year after the dam was built, the particles within the dam had probably already reached a reasonably stable position.

The comparison of the average rates of settlement between Nantahala Dam and test 3 are surprising. The average rate of settlement of the 2,000 pound increment, which represents about 65 to 70 feet of fill, was 0.0155 inches per day. The average rate of settlement of Nantahala Dam over a five-year period was 0.0147 inches per day. This was during the first five years after the dam was built. The time-settlement curve for Nantahala Dam is shown in Figure 10. The equation for this line on arithmetic graph paper is:

$$W = 1.278 (\log t) - .3$$

where W is in feet and t is in months.

The equation for the straight line portion of the 2,000 pound increment is:

$$W = 3 \times 10^{-4} (\log t) + .010$$

where W is in feet and t is in months, Figure 11. No immediate conclusion is drawn from these equations, but it is extremely interesting to note that the laboratory models (capped), like actual fills, settle at a constantly diminishing rate and at similar average rates.

Factors Influencing Settlement Data. Since the discussion here has been about average settlement, it is perhaps wise to realize the possibility of local settlement. When in dumping pieces of rock become lodged in such a manner that there are relatively large holes within the fill, then even the smallest adjustment resulting from the spalling of edges or points of contact may cause partial collapse of these voids and, hence, additional sudden settlement. This is known as local settlement and was noted in test 2 particularly where the action of the loading head crushed certain fragments more easily than others, thereby, causing the loading head to tilt. This differential settlement, of course, automatically averaged into the data because of the position of the dial gage and prompted the different procedure used in test 3.

Although the existence of large openings within the fill is probably largely a matter of chance, the shape of the pieces of rock has an important influence. For the rocks of the more spherical shape,

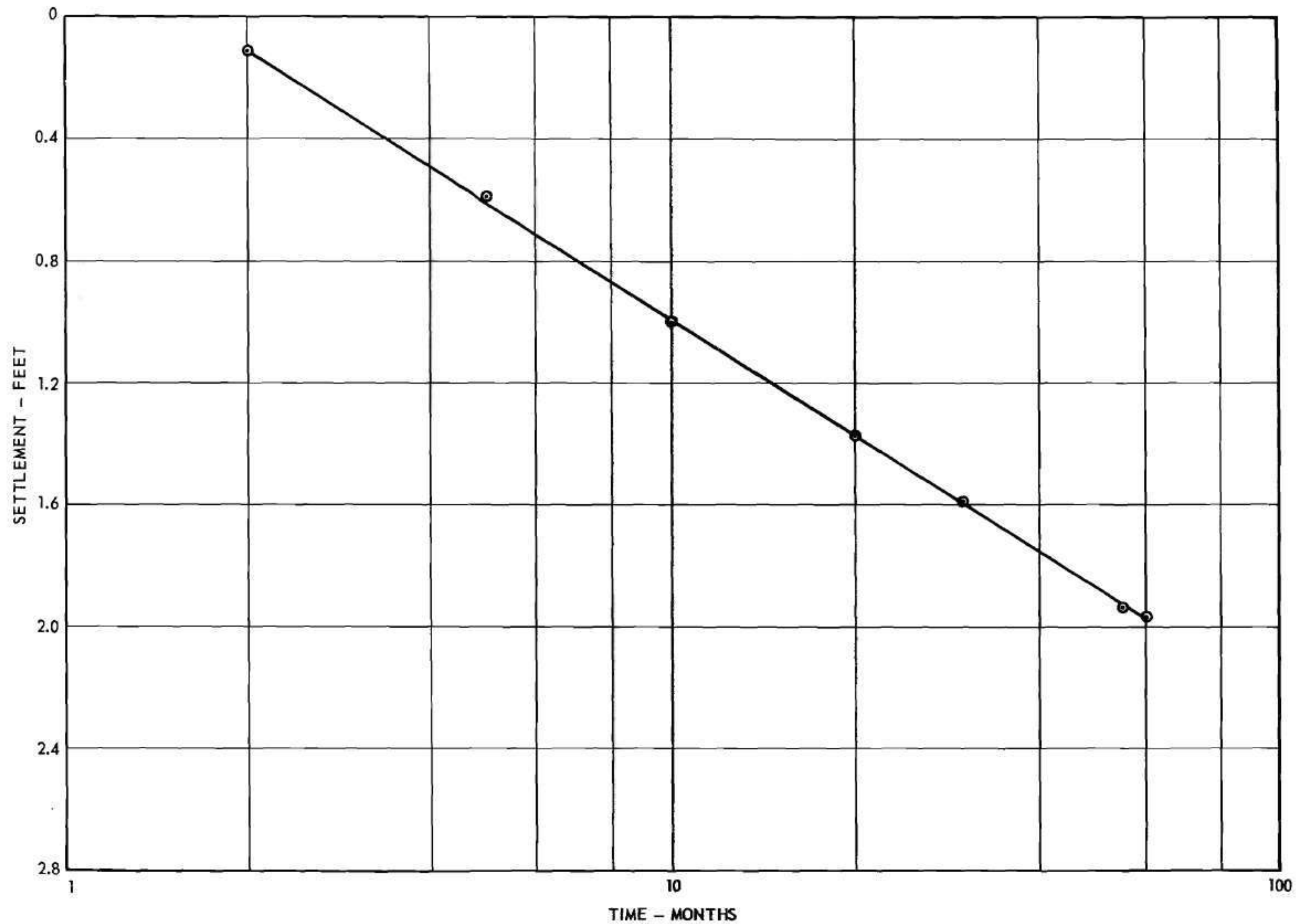


Figure 10. Time-Settlement Curve for Nantahala Dam for First Five Years After Construction.

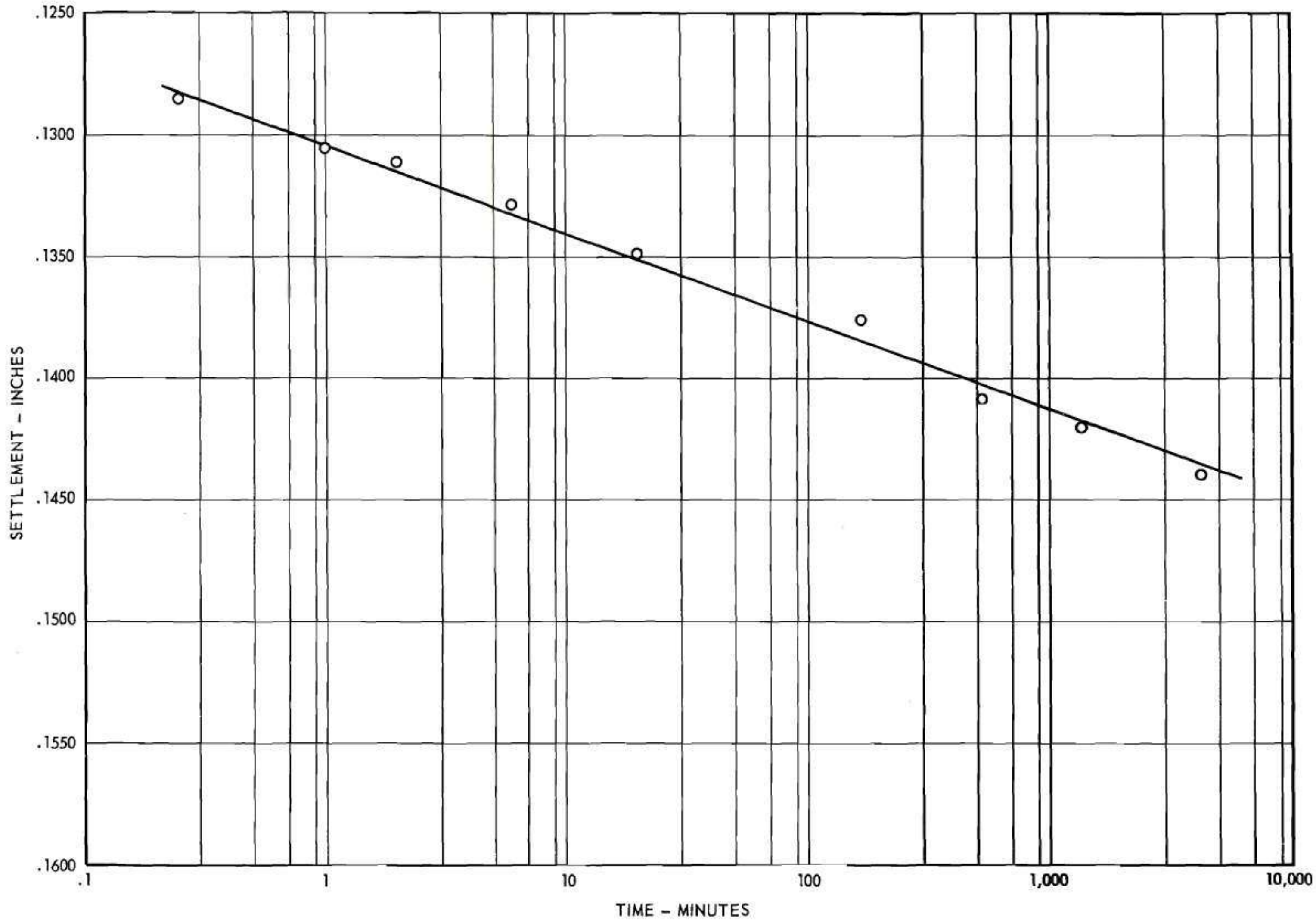


Figure 11. Straight-Line Portion of Time-Settlement Curve for 6500 psf Load Increment in Test 3.

the chances of occurrence of extra-large voids are materially less than for rocks of a flat, elongated shape, simply because they have fewer extremities upon which stresses can be concentrated thus allowing less fracturing.

CHAPTER IV

CONCLUSIONS

(1) The settlement of rockfills is due to the spalling and breaking of the points of contact and any finer material trapped between these points.

(2) The settlement of these fills continues at a gradually diminishing rate with time.

(3) When a rockfill material is tested in a normal consolidometer using normal loading procedures with smooth cap, the results are not indicative of the action within the actual fill.

(4) Local settlement is more prevalent in flat, elongated rocks. One explanation of the benefit of sluicing in reducing the amount of settlement after the construction of a rockfill, is that wetting lowers the rocks' unconfined compressive strength to its minimum value during construction.

(6) The compressive strength and angularity of the rock itself play an important role in the rock's resistance to settling.

(7) The settlement of large rockfills continues for many years after construction but is usually negligible after the first few years.

CHAPTER V

RECOMMENDATIONS

(1) Experiments should be undertaken to observe directly the interior of a rockfill during construction and subsequent application of water load.

(2) A parallel series of tests should be performed under both dry and saturated conditions.

(3) A standard procedure for the consolidation testing of rockfills should be developed and adopted.

(4) Additional work should be undertaken toward developing a theory for the time rate of settlement of rock fills.

A P P E N D I X

Table 1. Summary of Grain-Size Analyses

Sieve Number or Size	Weight of Material in Grams		
	Test 1	Test 2	Test 3
1"	2208	3252	2751
3/4"	570	196	151
1/2"	176	158	61
3/8"	58	41	17
No. 4	60	31	14
Pan	82	24	5
Total	3154	3702	2999

Table 2. Data from Consolidation Test 1.

Load on Sample (lbs.)	Elapsed Time from Application of Load (min.)	Deflection of Sample for Each Load (in 1/10,000 inches)
0	0.0	0
500	300.0	296
1000	420.0	457
2000	660.0	756
3000	180.0	251
4000	3360.0	461
5000	0.3	39
5000	0.5	41
5000	1.0	48
5000	2.0	79
5000	5.0	110
5000	10.0	111
5000	15.0	121
5000	30.0	130
5000	60.0	139
5000	120.0	151
5000	720.0	169
6000	0.25	61

Table 2. Data from Consolidation Test 1. (Continued)

Load on Sample (lbs.)	Elapsed Time from Application of Load (min.)	Deflection of Sample for Each Load (in 1/10,000 inches)
6000	0.50	390
6000	1.00	451
6000	2.00	491
6000	5.00	499
6000	15.00	509
6000	30.00	509
6000	60.00	521
6000	120.00	531
6000	240.00	533
6000	360.00	537
6000	690.00	539
10,000	0.25	512
10,000	.50	567
10,000	1.00	652
10,000	2.00	702
10,000	5.00	717
10,000	15.00	737
10,000	30.00	749
10,000	60.00	761
10,000	120.00	785
10,000	3480.00	917

Table 3. Data from Consolidation Test 2.

Load on Sample (lbs.)	Elapsed Time from Application of Load (min.)	Deflection of Sample for Each Load (in 1/10,000 inches)
0	0	0
500	0.25	179
500	0.50	198
500	1.00	201
500	2.00	206
500	5.00	212
500	10.00	217
500	20.00	221
500	30.00	222
500	60.00	226
500	120.00	229
500	240.00	231
500	480.00	232
500	1020.00	237
500	1260.00	239
500	2460.00	244
500	3210.00	248
500	3900.00	248
1000	0.25	160
1000	0.50	163

Table 3. Data from Consolidation Test 2. (Continued)

Load on Sample (lbs.)	Elapsed Time from Application of Load (min.)	Deflection of Sample for Each Load (in 1/10,000 inches)
1000	1.00	168
1000	2.00	172
1000	5.00	174
1000	10.00	179
1000	20.00	181
1000	30.00	183
1000	60.00	187
1000	165.00	196
1000	240.00	200
1000	465.00	205
1000	902.00	208
1000	1620.00	251
1000	2130.00	255
1000	3600.00	256
1000	4460.00	256
1000	5760.00	256
2000		0
2000	.25	207
2000	.50	209
2000	1.00	216
2000	2.00	224

Table 3. Data from Consolidation Test 2. (Continued)

Load on Sample (lbs.)	Elapsed Time from Application of Load (min.)	Deflection of Sample for Each Load (in 1/10,000 inches)
2000	5.00	264
2000	10.00	276
2000	20.00	281
2000	30.00	287
2000	60.00	295
2000	120.00	308
2000	240.00	315
2000	360.00	317
2000	480.00	320
2000	720.00	321
2000	840.00	322
2000	1340.00	336
2000	1620.00	368
2000	2836.00	376
2000	3300.00	381
2000	4320.00	384
2000	4860.00	384
2000	5760.00	384
5000		0
5000	.25	734
5000	.5	773

Table 3. Data from Consolidation Test 2. (Continued)

Load on Sample (lbs.)	Elapsed Time from Application of Load (min.)	Deflection of Sample for Each Load (in 1/10,000 inches)
5000	1.00	803
5000	2.00	824
5000	3.00	840
5000	5.00	890
5000	10.00	912
5000	20.00	932
5000	30.00	940
5000	60.00	951
5000	120.00	961
5000	240.00	1035
5000	480.00	1049
5000	1500.00	1052
5000	1980.00	1053
5000	4320.00	1057
5000	5040.00	1060
5000	5160.00	1062
5000	5760.00	1073
5000	6060.00	1074
5000	7210.00	1080
5000	7910.00	1080
5000	8650.00	1080

Table 3. Data from Consolidation Test 2. (Continued)

Load on Sample (lbs.)	Elapsed Time from Application of Load (min.)	Deflection of Sample for Each Load (in 1/10,000 inches)
10,000		0
10,000	.25	1587
10,000	.50	1674
10,000	1.00	1714
10,000	2.00	1820
10,000	3.00	1871
10,000	5.00	1920
10,000	10.00	1954
10,000	20.00	1974
10,000	30.00	1984
10,000	60.00	2002
10,000	120.00	2013
10,000	240.00	2033
10,000	480.00	2049
10,000	1232.00	2054
10,000	1711.00	2064
10,000	4300.00	2076
10,000	5640.00	2084
10,000	6500.00	2087
10,000	7460.00	2090
10,000	7580.00	2102

Table 3. Data from Consolidation Test 2. (Continued)

Load on Sample (lbs.)	Elapsed Time from Application of Load (min.)	Deflection of Sample for Each Load (in 1/10,000 inches)
10,000	7700.00	2132
10,000	7880.00	2150
10,000	7940.00	2153
10,000	8650.00	2292
10,000	8800.00	2324
10,000	8980.00	2349
10,000	9230.00	2363
10,000	9520.00	2370
10,000	10,000.00	2371

Table 4. Data from Consolidation Test 3.

Load on Sample (lbs.)	Elapsed Time from Application of Load (min.)	Deflection of Sample for Each Load (in 1/10,000 inches)
500	0	0
500	.25	550
500	.50	568
500	1.00	575
500	2.00	589
500	4.00	595
500	5.00	600
500	11.00	609
500	15.00	614
500	20.00	618
500	35.00	625
500	50.00	635
500	62.00	639
500	111.00	648
500	120.00	648
500	195.00	658
500	255.00	660
500	292.00	665
500	619.00	687
500	1410.00	694
500	1880.00	703

Table 4. Data from Consolidation Test 3. (Continued)

Load on Sample (lbs.)	Elapsed Time from Application of Load (min.)	Deflection of Sample for Each Load (in 1/10,000 inches)
500	2880.00	711
500	3300.00	719
500	4320.00	721
1000	.25	138
1000	.50	140
1000	1.00	147
1000	2.00	150
1000	3.00	154
1000	7.00	160
1000	11.00	165
1000	15.00	169
1000	30.00	178
1000	60.00	188
1000	120.00	198
1000	189.00	207
1000	240.00	210
1000	480.00	227
1000	548.00	232
1000	720.00	236
1000	1560.00	246
1000	2190.00	249

Table 4. Data from Consolidation Test 3. (Continued)

Load on Sample (lbs.)	Elapsed Time from Application of Load (min.)	Deflection of Sample for Each Load (in 1/10,000 inches)
1000	3060.00	252
1000	3760.00	255
1000	5065.00	259
1000	5791.00	260
2000	0	0
2000	.25	304
2000	.50	317
2000	1.00	324
2000	2.00	330
2000	3.00	339
2000	6.00	347
2000	10.00	354
2000	20.00	368
2000	49.00	378
2000	62.00	380
2000	83.00	385
2000	164.00	395
2000	286.00	408
2000	388.00	411
2000	538.00	427
2000	600.00	429

Table 4. Data from Consolidation Test 3. (Continued)

Load on Sample (lbs.)	Elapsed Time from Application of Load (min.)	Deflection of Sample for Each Load (in 1/10,000 inches)
2000	780.00	432
2000	1380.00	439
2000	1718.00	449
2000	3000.00	456
2000	4229.00	456
5000	0	0
5000	.25	683
5000	.50	695
5000	1.00	704
5000	2.00	713
5000	4.00	721
5000	8.00	725
5000	12.00	731
5000	17.00	732
5000	26.00	734
5000	49.00	743
5000	60.00	745
5000	127.00	755
5000	130.00	761
5000	360.00	769
5000	1380.00	785

Table 4. Data from Consolidation Test 3. (Continued)

Load on Sample (lbs.)	Elapsed Time from Application of Load (min.)		Deflection of Sample for Each Load (in 1/10,000 inches)
5000	2185.00	Wet sample	813
5000	2191.00		835
5000	2193.00		846
5000	2197.00		874
5000	2200.00		938
5000	2201.00		968
5000	2210.00		1023
5000	2217.00		1038
5000	2219.00	Drained sample	1044
5000	2221.00		1061
5000	2274.00		1069
5000	2284.00		1071
5000	2850.00		1103
5000	3798.00		1155
5000	7080.00		1193
10,000	0		0
10,000	.25		1122
10,000	.50		1172
10,000	1.00		1222
10,000	2.00		1243

Table 4. Data from Consolidation Test 3. (Continued)

Load on Sample (lbs.)	Elapsed Time from Application of Load (min.)	Deflection of Sample for Each Load (in 1/10,000 inches)
10,000	4.00	1267
10,000	7.00	1281
10,000	16.00	1303
10,000	22.00	1314
10,000	36.00	1340
10,000	77.00	1403

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